HAYS

Design of a Cantilever Bridge

Civil Engineering
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DESIGN

OF A

CANTILEVER BRIDGE

BY

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THESIS

FOR

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IN

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THIS IS TO CERTIFY THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

Carl Hays

ENTITLED Design of a Cantilever Bridge

IS APPROVED BY ME AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE DEGREE

OF Bachelor of Science in Civil Engineering.

Ira O. Baker.

HEAD OF DEPARTMENT OF Civil Engineering.

DESIGN OF A CANTILEVER BRIDGE

INTRODUCTORY

It is proposed to design a cantilever bridge to take the place of the highway bridge spanning the Vermillion river at the foot of Vermillion Street, Danville, Illinois. It is reported that at the time the bids were received for the present structure, some bidders proposed to build a cantilever bridge. The writer proposes to design a cantilever bridge for this location, partly to familiarize himself with the principles of the cantilever and partly to make a comparison of the relative economy of the two forms of bridges. The subject will be treated as a general design only, with special attention to economy and to those points which are foreign to common bridge designing.

The present bridge was erected in 1893 by the La Fayette Bridge Company, and consists of two 220 ft. spans with steel trestle approaches. The entire length is about 1050 ft. There is a pier in the center of the stream which is objectionable,

since it obstructs the current. To reduce this objection the pier is oblique to the bridge, which in turn makes a skew panel in each of the 220-ft. spans. Both of these objections will be removed by a cantilever bridge.

The cantilever was adopted on account of its clear water-way and the ease of its erection. The only serious objection to this form of bridge is the excessive vibration produced by the live load, but special care will be taken to reduce this vibration.

The bridge will be designed by Cooper's Specifications for Highway Bridges, and on the points in which these specifications are deficient, the Specifications for the Cincinnati-Newport Cantilever Bridge will be consulted. The bridge will be designed to carry the suburban traffic of a city of 40000 inhabitants.

The bridge will be investigated for an electric railway, which uses 12-ton cars having a length of 25 ft. center to center of couplings. In addition, a live load of 80 pounds per square foot will be assumed, as the neighborhood is rich in bituminous coal and a coarse grade of limestone. A bridge designed for this loading will be very heavy; and will be excessive for the

present, as the actual maximum load will not be equal to that assumed. As the load carried by the vehicles increase with the growth of civilization, and as the amount of traffic increases with the growth of business in the growing city, the assumed load may not be excessive for any considerable length of time

PROPORTIONS OF THE SUPERSTRUCTURE

LENGTH. The center or channel span was assumed to be 600 ft., which gives ample water-way. From this the lengths of the cantilever arms and suspended span were found by a method to be explained later. With this length of channel span and proper lengths of shore arm and suspended span, the bridge will approximately span from bluff to bluff, and no steel approaches will be necessary.

Suspended Span. The economic length of the suspended span was taken as $\frac{5}{12}$ of the main span, as proven by Prof. S. W. Robinson in his investigation of the Red Rock Cantilever Bridge. He found (1) that for any suspended span there was no difference in economy if the allowable stress per square inch was constant; (2) that for the

formula $s\left(1 + \frac{min}{max}\right)$, there was no difference in economy between $\frac{1}{3}$ and $\frac{1}{2}$ the length of the main span; (3) that the lower the maximum unit stress, the smaller the ratio between the suspended and center spans; and (4) that for an allowable unit stress of $1 + \frac{\text{minimum total stress}}{\text{maximum total stress}}$, he found there was a saving of 1.6 per cent for suspended spans of $\frac{5}{12}$ to $\frac{1}{2}$ the length of the main span. He then concludes that a ratio of $\frac{5}{12}$ is the most economic relation between the suspended and center spans.

Shore Cantilever Arm. To find the most economic length of the shore cantilever arm, a length of 250 ft. was first tried. With this length and the approximate dead and live loads, the stresses were found graphically. The reactions at the end of the shore arm were found by taking moments of one half the bridge about the center pier. This reaction was 57 tons up for the dead load and 144 tons up for the live load. With these reactions, the stresses in the chords are greater than those of the Red Rock Cantilever, which has a longer span, was designed for a much heavier load, has a shallower depth, and has chords at a less inclination. The stresses in the bridge being designed were considered to be too great for economy, and

therefore a longer arm was assumed.

By trial it was found that a length of shore arm of 300 to 325 ft. would nearly balance the negative moment of the river arm and half of the suspended span. Since a length of 325 ft. gives a reaction nearly zero, and gives a more uniform upper chord, that length of the shore cantilever arm was chosen. The lengthening of the shore arm greatly reduced the stresses in the chords, changing that in the upper chord from an excessive tension to a medium compression, and that of the lower chord to a small tension. A length of shore arm of 325 ft. was therefor considered to be the best length to employ.

DEPTH. Suspended Truss and Approaches. The depth of the suspended truss and of the entrance to the shore arm was assumed as 30 ft. center to center of chord pins. This allows 5 ft. for the floor system, 5 ft. for the portal and sway bracing, and leaves a clearance of 20 ft. This clearance was considered sufficient for street-cars and the highest loads. This depth with a panel length of 25 ft. gives an angle between the webs and the verticals of about 40° which is approximately the economic angle.

Over Center Piers. The depth over the

center piers should be such as will give an angle of about 40° between the posts and the web members. With double intersection web members and a panel length of 25 ft., this depth was found to be 105 ft. The lower chord pin was depressed 25 ft. below the line of the pins in the suspended span and in the end of the shore arm, while the upper pin is 80 ft. above this line. The points where the lateral rods meet the upper and lower chords are located on parabolas. The parabolic upper chord extends over both cantilever arms, but the lower chord does not continue as a parabola to the shore arm pier, but is partly on the line of the pins in the suspended truss and in the approaches.

WIDTH OF BRIDGE. The driveway will be designed to carry a street railroad in the center with a clear space of 10 ft. on each side. This gives a roadway 28 ft. in the clear; and assuming the trusses as 3 ft. wide (thick), the distance from center to center of trusses is 31 ft. Two sidewalks 6 ft. in width will be supported on cantilevers outside of the trusses.

The rule for ordinary bridges that the width should be $\frac{1}{18}$ of the length center to center of end pins does not hold in this case. Robinson proved for the Red Rocks Cantilever Bridge that

$\frac{1}{27}$ was as good a ratio for a cantilever bridge, as $\frac{1}{18}$ is for a simple bridge. In this case a ratio of $\frac{1}{20}$ of the main span is about right.

Width over Center Piers. Robinson found that the best ratio between the width of the bridge (center to center of trusses) to the width over the center pier is $n = \frac{Sl}{Eb}^2 \left(\frac{\frac{5}{12}}{2}\right)^2$, in which b = width of the suspended span, E = 28000000 lbs., S = allowable unit stress which was assumed as 12,500 lbs., for a mean of both tension and compression, l = length of the center span, and n = ratio of the width of the suspended span and the width over the center pier. By this formula the width (center to center of lower chord pins) over the center pier would be 38+ ft or say 39 ft. This formula for the economic width would require a gradual widening, from 31 to 39 ft., from the shore arm and also from the suspended span to the center pier. This was not done for two reasons; (1) the varying width would cause an extra complication in the shop work and in the assembling in the field; and, (2) the horizontal thrust caused by the laterals expanding and contracting would have a tendency to crack the pier and thus impair the bridge.

PANEL LENGTH. Short panel lengths are not as necessary now as formerly, since steel is comparatively cheap. Further, the panel length in good practice is continually increasing, therefore a panel length of 25 ft.

was assumed. This length is long for ordinary highway bridges. The panel length in the Cincinnati-Newport bridge is between 26 and 27 ft., which at the time (1890-91) was considered excessive.

LOADS. Dead Load. The dead load per ft. will vary with the depth, and therefore will decrease regularly from the center pier outward, the curve of variation being a parabola, with depths and relative loadings for abcissas and ordinates respectively. Since the weight decreases from the center pier outward, the reaction at the end of the shore arm, and consequently the stresses in the cantilever arms will be less than if the load was assumed to be uniform.

The Cincinnati-Newport cantilever bridge has a clear roadway of 26 ft., with two 7 ft. sidewalk cantilevers at each side. The trusses occupy 6 ft. of width, making the entire width of the bridge 46 ft. The average weight of bridge is 1.53 tons per foot of bridge. Reasoning from this bridge, and assuming the weight to vary as the width and as the square of the span, it is concluded that the uniform dead load of the proposed bridge will be 1.7 tons per foot of bridge, or 21.25 tons per joint per truss. One third of the dead load will be considered as acting at the upper chord joints.

Live Load. The live load will be considered

as 80 lbs. per square foot of roadway and sidewalk, which makes a joint load of 20 tons per truss. The stresses due to the live load were found for two conditions; (1) the shore arm loaded; and, (2) the river arm and one half the suspended span loaded. The live load stresses in the inclined web members were found algebraically. The bridge will also be investigated for a track standing full of motor cars weighing 12 tons on 25 ft.

Wind Load. The wind pressure will be considered as 30 lbs. per square foot of exposed surface, and will be treated both as a live and a dead load.

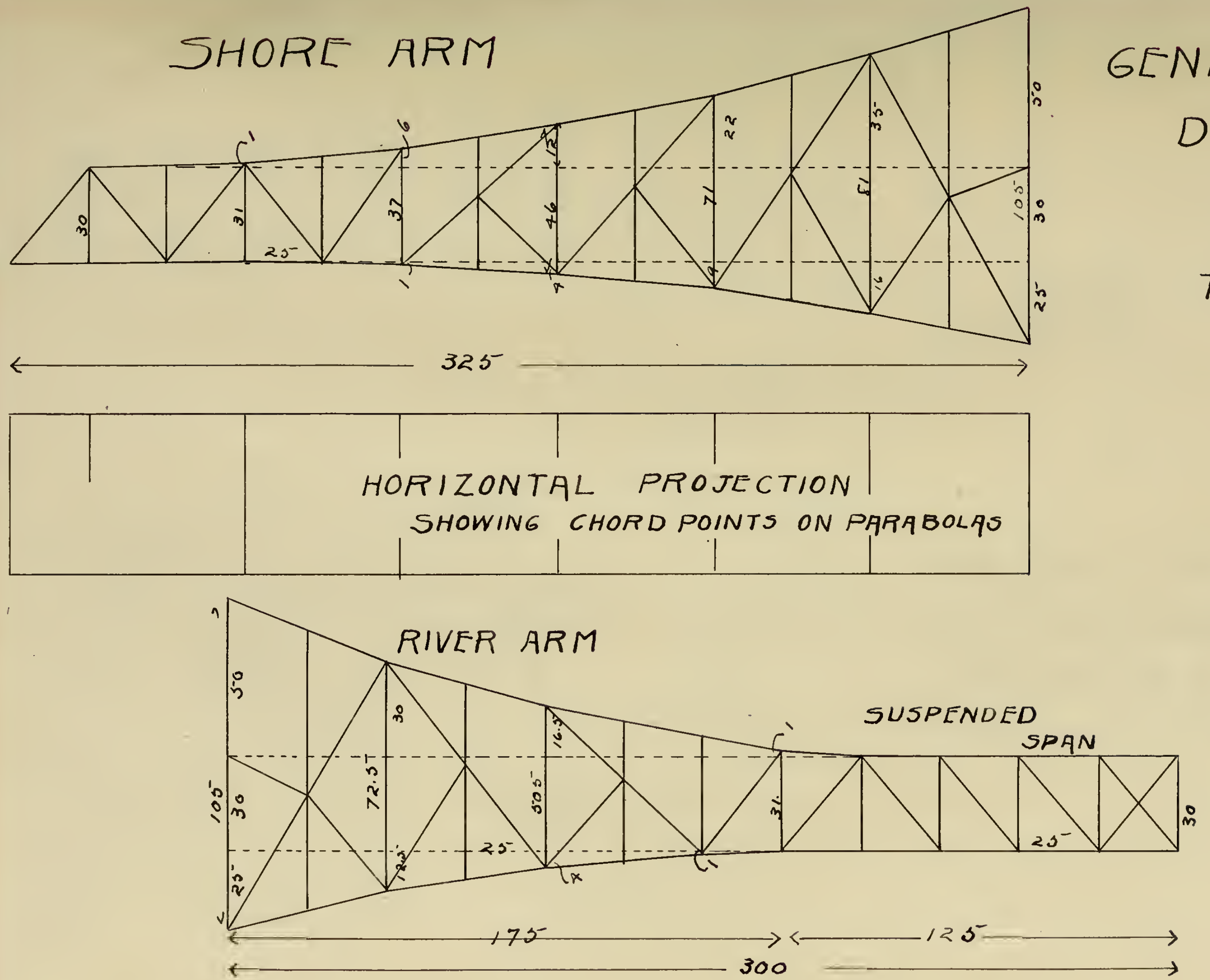
On Dead Load. There is no way of finding the amount of surface exposed to the wind by the truss, except from the drawings of existing bridges. The drawings of the St. Lawrence River Cantilever Bridge, — a railroad bridge 841 ft long—was used, from which the approximate area per foot was found to be from 12 to 15 square feet for different parts of the structure. The wind area varies with the depth of the truss being greatest at the center piers and least in the suspended span. A uniform wind pressure would be in excess of the correct pressure for the suspended span; while in the cantilever arms this pressure would be smaller than the actual. The assumed pressure in the suspended span being greater than the actual, produces

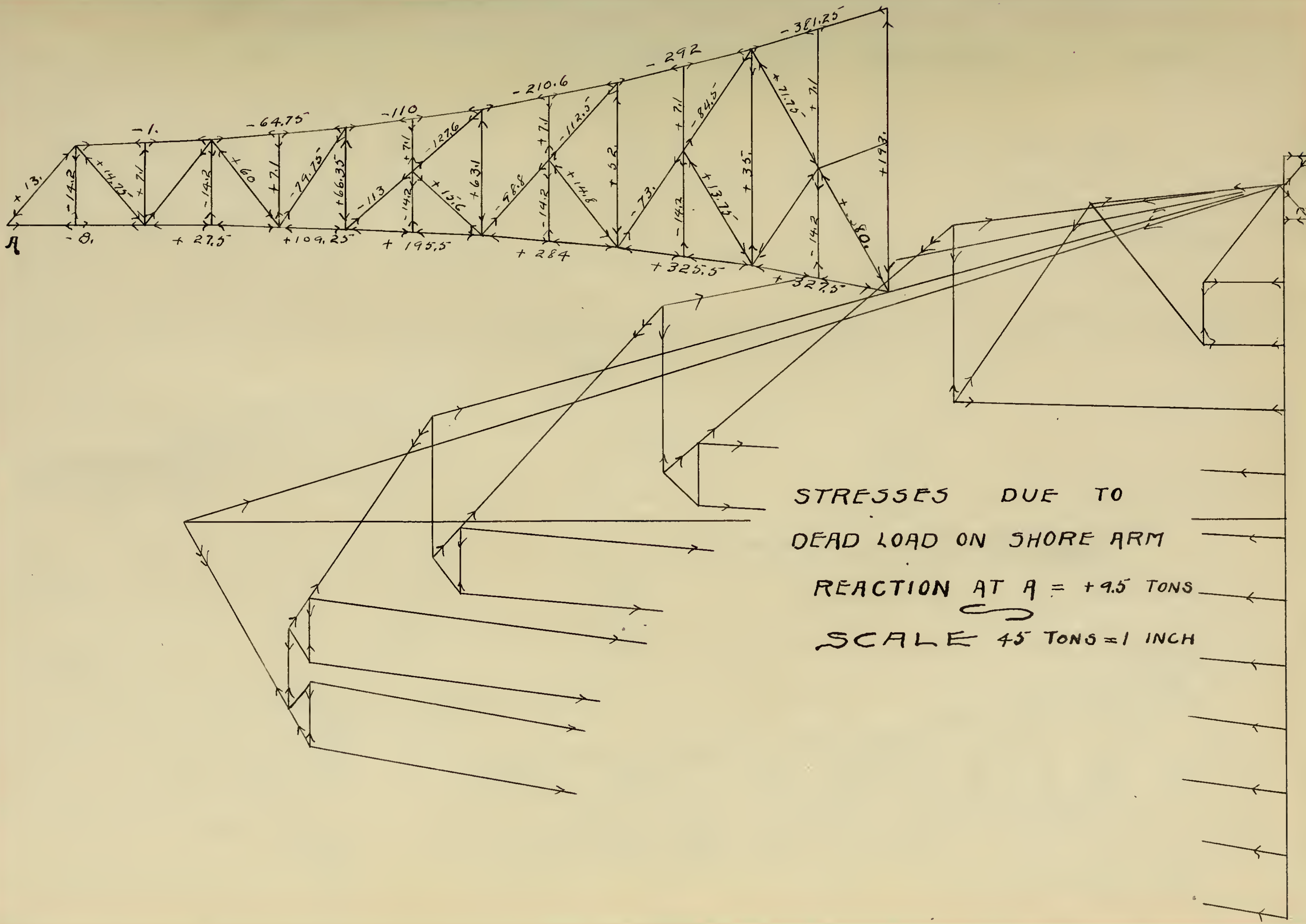
excessive chord stresses in that part of the structure; while in the cantilever arms the assumed being smaller than the actual pressure, causes a diminution of the chord stresses. Hence the assumption of a uniform pressure tends to equalize the wind stresses in the chords. It was therefore decided to use a uniform wind pressure, and assume the exposed area as being approximately the same as the mean of the St. Lawrence River Bridge, ie; $13\frac{1}{3}$ square feet per linear foot. This makes the wind pressure on the truss equal to 4000 lbs. per linear foot of bridge, half of which will be considered as concentrated at the panel points of each chord. This assumption is not strictly correct, since the concentration of the area due to the stringers, fence, and the lower compression struts would have a tendency to carry more than one half the wind load on the truss to the lower chord joint.

On Live Load. If a train of electric cars extends from end to end of the bridge, the wind pressure will be about 150 lbs. per linear foot; but as this condition will occur very rarely, it will be assumed that the maximum wind pressure on the live load is 100 lbs. per linear foot. This assumed wind pressure of 100 lbs., although excessive, will be used; so as to take account of the maximum concentration on any part of the bridge.

Weight of traveller. The traveler, including ballast, was figured as weighing 40 tons. The maximum stresses in the chords due to the traveler will occur when it is in the center of the suspended span. The maximum stresses in the tension diagonals will occur when the traveler is at the point where the diagonal intersects the lower chord. The maximum stress in the compression diagonal will occur when the tension member that meets it in the upper chord is at its maximum. The traveler will cause a reversal of stresses in the chords of the suspended span, and therefore those members will have to be designed with this object in view.

GENERAL DIMENSIONS OF TRUSSES





STRESSES DUE TO

DEAD LOAD ON SHORE ARM

REACTION AT A = +9.5 TONS

SCALE 45 TONS = 1 INCH

LIVE LOAD STRESSES IN SHORE ARM
DUE TO
LIVE LOAD ON SHORE CANTILEVER ARM

SCALE 60 TONS = 1. INCH

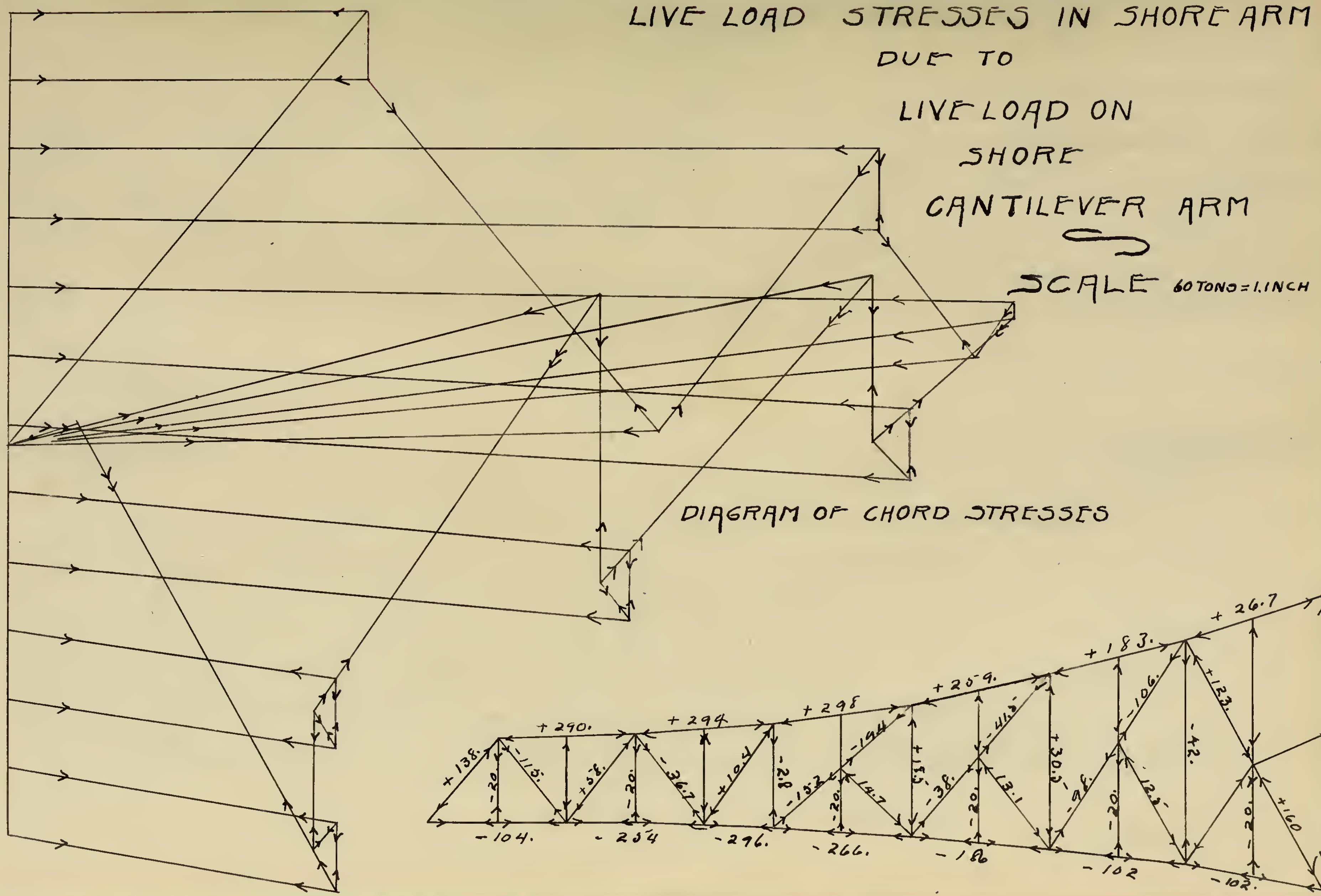
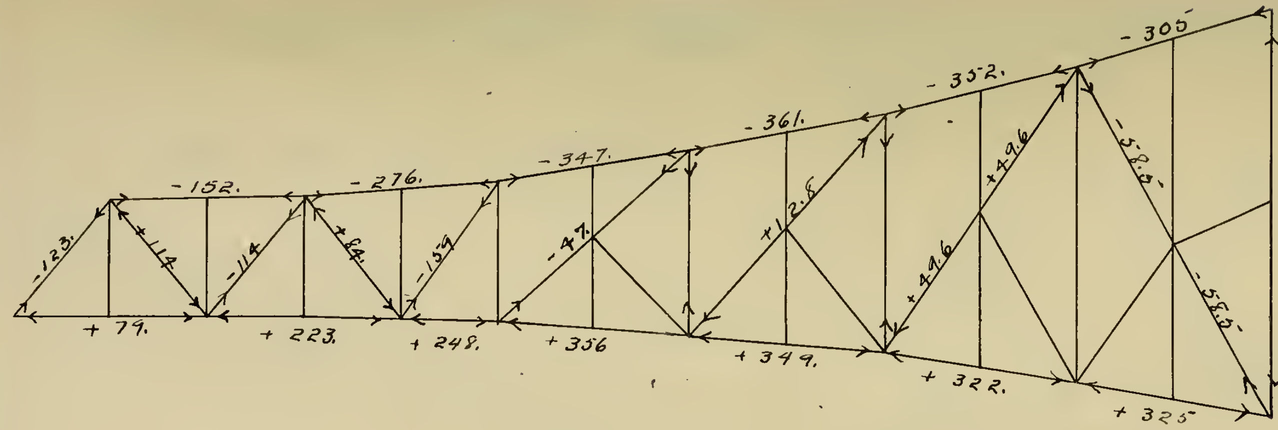


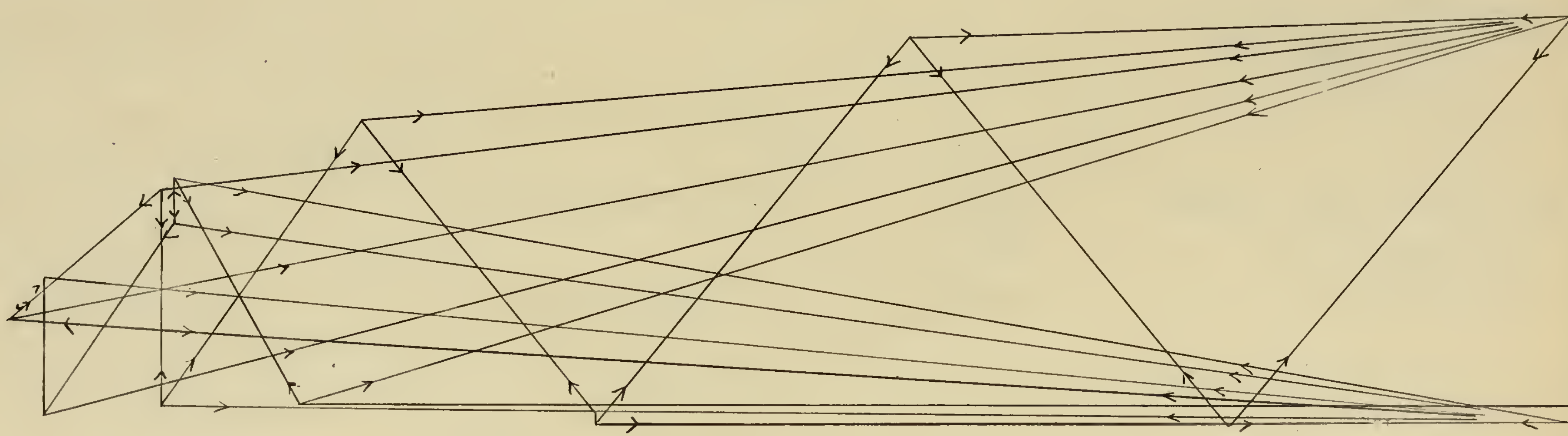
DIAGRAM OF CHORD STRESSES

STRESSES IN SHORE ARM
DUE TO
LIVE LOAD ON RIVER ARM
OF CANTILEVER AND
ONE HALF SUSPENDED
SPAN

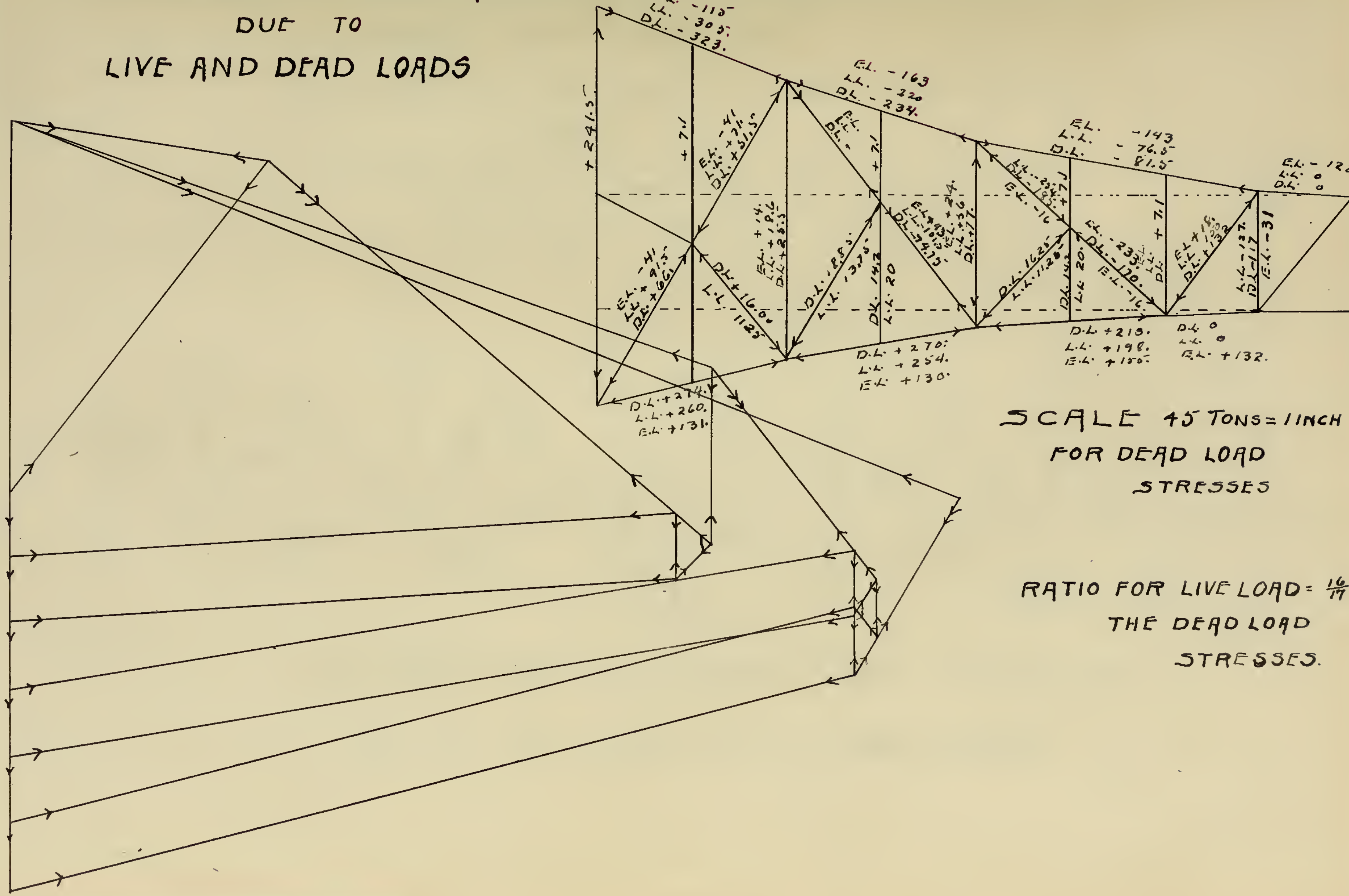
LIVE LOAD 80 LBS. PER □ FT.

SCALE 40 TONS = 1 IN.





STRESSES IN RIVER ARM
DUE TO
LIVE AND DEAD LOADS



SCALE 45 TONS = 1 INCH
FOR DEAD LOAD
STRESSES

RATIO FOR LIVE LOAD = $\frac{16}{17}$
THE DEAD LOAD
STRESSES.

MAXIMUM AND MINIMUM STRESSES

IN SHORE CANTILEVER ARM

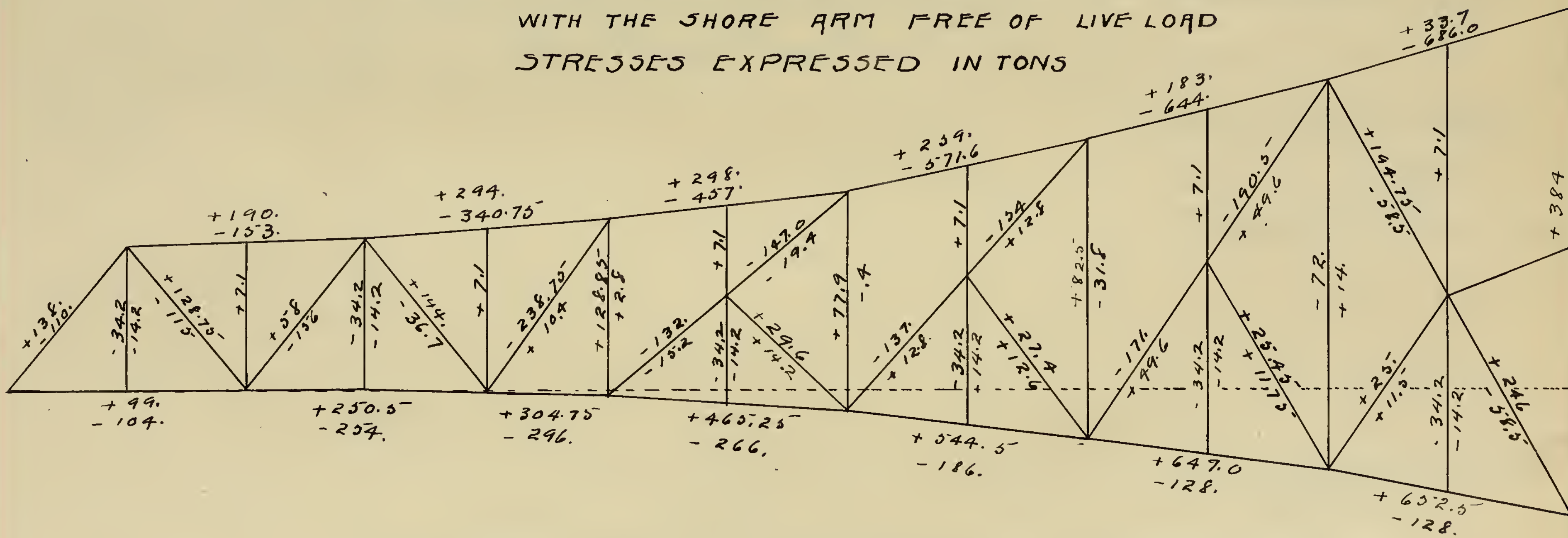
DUE TO LIVE AND DEAD LOADS ON SHORE CANTILEVER ARM

AND

LIVE AND DEAD LOADS ON RIVER CANTILEVER ARM AND SUSPENDED SPAN

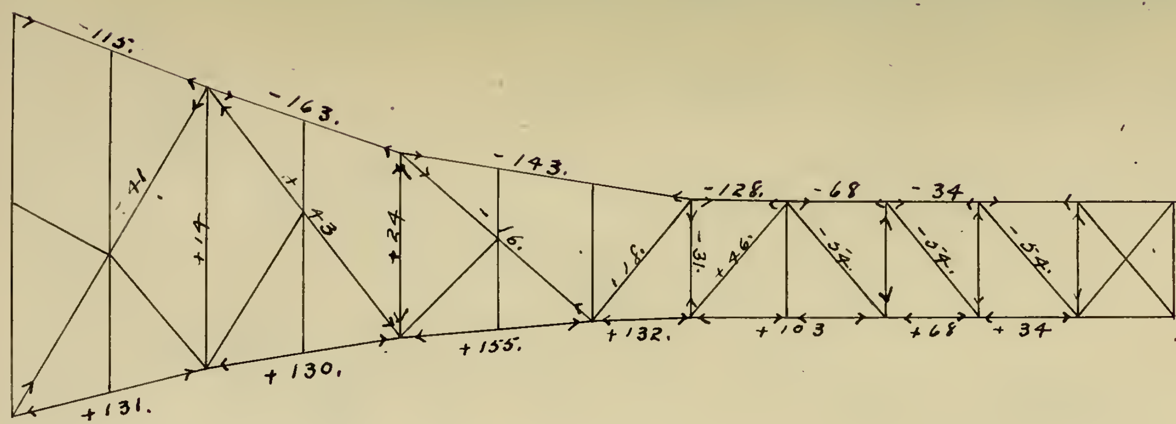
WITH THE SHORE ARM FREE OF LIVE LOAD

STRESSES EXPRESSED IN TONS



ALL POSTS WILL BE DESIGNED FOR COMPRESSION

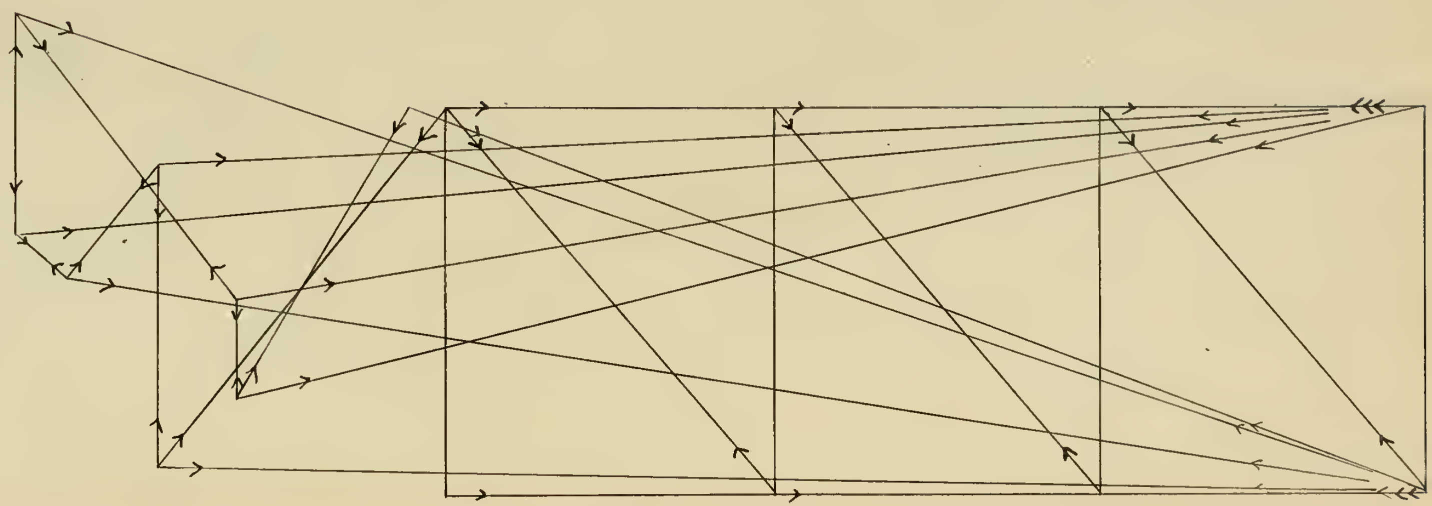
BELOW THE PLANE OF THE FLOOR BEAMS



ERECTION STRESSES
DUE TO
TRAVELER
IN CENTER OF THE
SUSPENDED SPAN

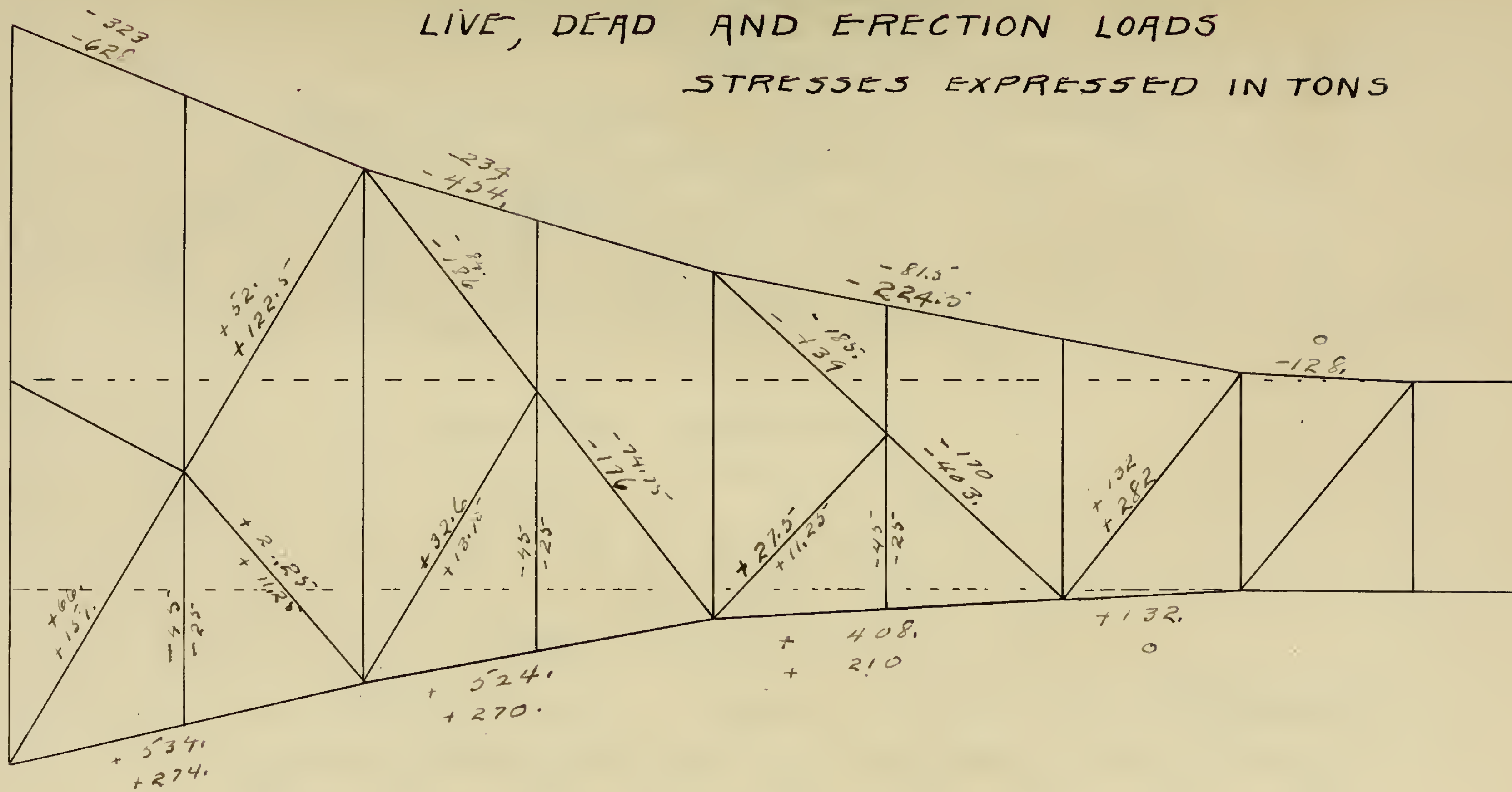
WEIGHT 40 TONS

SCALE 20 TONS = 1 INCH



MAXIMUM AND MINIMUM STRESSES IN RIVER ARM
DUE TO
LIVE, DEAD AND ERECTION LOADS

STRESSES EXPRESSED IN TONS



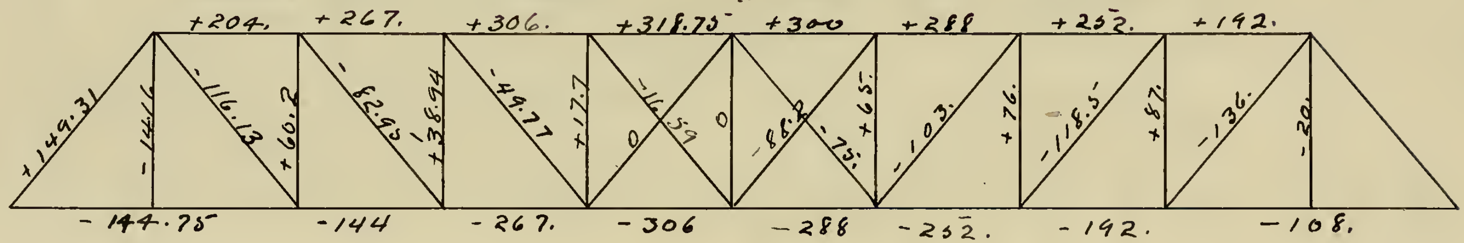
STRESSES IN SUSPENDED SPAN

JOINT DEAD LOAD = 21.25 TONS

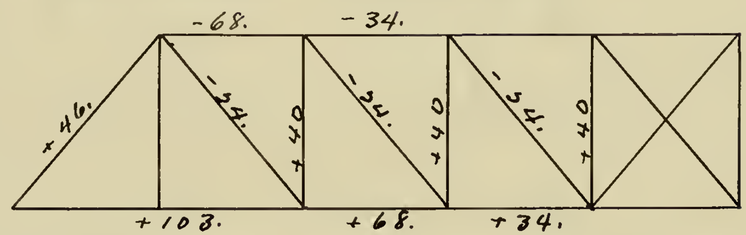
LIVE LOAD = 20 TONS

SEC. = 1.562 , TAN = 1.2

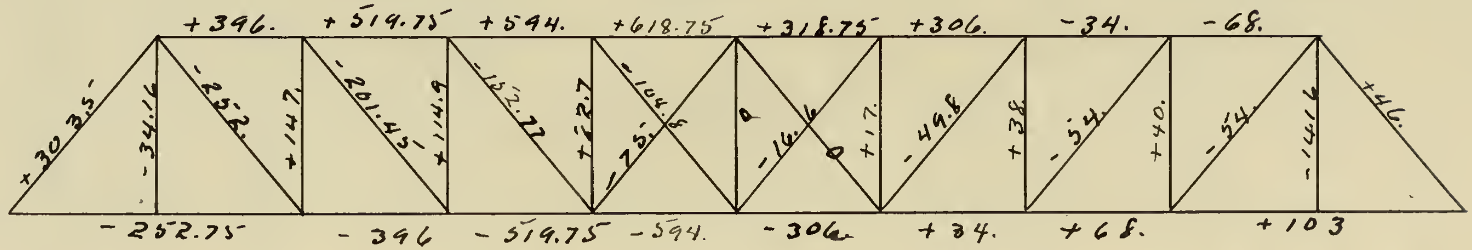
< Dead Load Stresses >< Live Load Stresses >



ERECTION STRESSES



MAXIMUM AND MINIMUM STRESSES



DESIGN OF STRINGERS

1. Under Sidewalk.

Wt. of flooring per stringer = $\frac{6'}{3} \times 25' \times 2\frac{1}{2}'' \times 4\frac{1}{4}^{\#}$ = 550#
Live Load per stringer = $80^{\#} \times 25' \times 2'$ = 4000#
Weight of stringer estimated = 750#
Total load per stringer = 5300#

a 9" - 25#- I @ 12500 lbs. per sq. in. will carry 6290 lbs. which is safe and will be employed.
7/8" rivets will be used, of which the double shear is 4100#. The total number of rivets required is 3, but the number used in each angle will be 3. Two 4" x 4" x ½" hitch angles will be used at each end.

2. Under Roadway. (exclusive of street car tracks).

Wt. of flooring = $2\frac{1}{2}'' \times 25' \times 5'' \times 4\frac{1}{4}^{\#}$ = 1310#
Live Load = $80 \times 25' \times 2\frac{1}{2}'$ = 5000#
Wt. of stringer estimated = 800#
Total load per stringer = 7110#

a 10" - 25# - I @ 12500 lbs. per sq. inch will carry 8940 lbs, and will be used.
The riveting will be the same as in the sidewalk stringers, and hitch angles similar.

3. Under Railway. (There will be one stringer under each rail and one under the middle of the track).

Wt. of ties = $15 - 6'' \times 8'' \times 14' \times 4\frac{1}{4}^{\#}$ = 3000#
Wt. of rails = 80×25 = 2000#
Wt. of flooring = same as in roadway = 1310#

Amount carried forward = 6310#

Wt. of stringer estimated = 1000#

Total dead-load per stringer = 7310#

Live Load = $\frac{2 \times 24000}{3}$ = 16000#

Total load per stringer = 23310#

A 20"- 65#. I @ 12500 lbs. per sq. in. will carry 24360# and will be employed. Rivets = $\frac{24360}{4100}$ = 6.
Connecting angles used = 4"x4"x½".

Weight of Stringers per Panel.

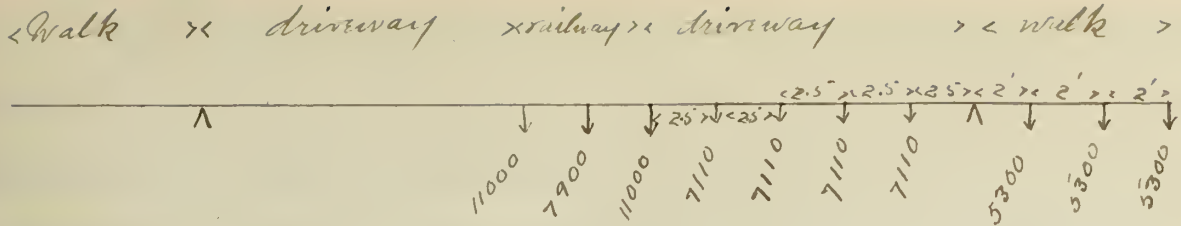
Name	Number	Length in ft.	Cross-Sect.	Wt. per ft.	Total Weight Main Mr.	Total Weight Details
I	6	25	9"	21	3150	
I	8	25	10"	25	5000	
I	3	25	20"	65	4890	
Angles	24.	.66	4"x4"x½"	12.8		203.
"	36	7.5	"	"		307
"	12	1.5	"	"		230
R. Heads	290	—	—	24.3 per C.		71
					810	810

Total weight = 13,850#

Weight of stringers for entire bridge = 692,500#.

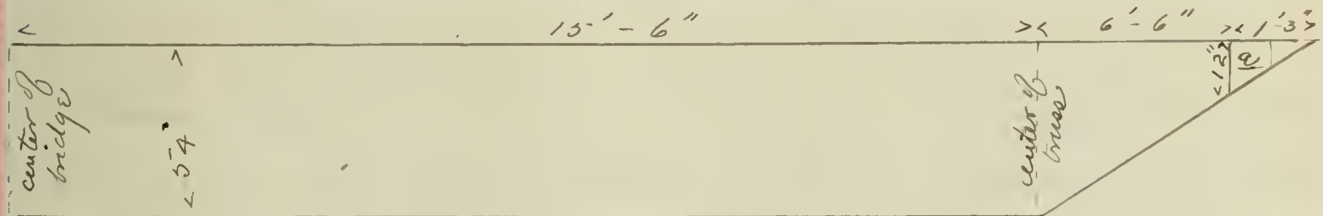
DESIGN OF FLOOR BEAMS

The floor beams will be built up members continuous between the trusses, with cantilevers at the two ends supporting the sidewalk. The amount and positions of the several loads are shown in the accompanying diagram.



The moment at the center due to the loads between the supports = 7900 + 5(15000) + 70(7110) = 580,600 ft. lbs. The negative moment of the loads on the cantilever arms = 53,000 ft. lbs. The total moment for the entire floor beam is 527,600 ft. lbs. or 6,324,000 inch lbs.

Economic Depth $= 1.41\sqrt{\dfrac{6324000}{12000\left(\frac{3}{8}\right)}} = 53.68$ inches. This is the economic depth at the center, but the depth will be made uniform between the trusses, on account of the irregular loading they will be subjected to. The cantilever arms will be tapered as shown below.



The minimum theoretical depth at a will not be used, because the ballustrade posts will be fastened to the web of the floor beam at this point, and extra depth will be needed for this purpose. The top and bottom flange angles will be continuous until they intersect, being here connected by a plate, and from this point a brace will be connected to the fence.

Flange Angles. Tension stress in flanges = $\frac{6324000}{54}$

$= 117500^{\#}$, which requires an area of 7.35 square inches. Two $4'' \times 6'' \times \frac{3}{8}''$ angles and a cover plate $12'' \times \frac{1}{4}''$ will be assumed of which the -

gross area of two angles	=	7.22 sq. in.
gross area of cover plate	=	3.00 sq. in.
total gross area	=	10.22 sq. in.
deduct for rivets	=	1.44 sq. in.
total net area	=	8.78 sq. in.

This area will be sufficient for the compression flange, as well as for the tension flange. The cover-plate will not be used on the cantilever arms, since the loads will not require it.

Riveting. The spacing in the flange angles at the center line of the truss was obtained by the formula $p = \frac{rh}{S} = \frac{(4400)54}{49900} =$ 5 inches. This spacing will continue for 8'-10", or to the second stiffener from the center line of the truss, after which the pitch will continue at 6 inches. The rivet spacing in the flange plate $= \frac{7.22 + 10.22}{7.22} =$ 2.4 inches for single, or 4.75 inches for double riveting. The spacing in the cantilever arms will not be figured but will be placed at 6 inches, which is the maximum spacing

Stiffeners. The stiffeners will be $3'' \times 3'' \times \frac{3}{8}''$ angles, uniformly spaced 53 inches apart, and with rivet spacing of 6 inches.

Weight of a Floor Beam.

Name	Number	Length in feet	Cross Sec.	Wt. per ft.	Total Weight Main Mr.	Details
Plate	1	31	$57'' \times \frac{3}{8}''$	72.6	2250	
"	2	7.5	$34.5'' \times \frac{3}{8}''$	49.	735	
"	1	31	$12'' \times \frac{1}{4}''$	10.2	322	
angles	2	46	$4'' \times 6'' \times \frac{3}{8}$	12.3	1130	
"	2	47.5	"	12.3	1170	
"	4	1	$4'' \times 4'' \times \frac{3}{8}''$	9.7		38
Stiffeners	14	4.5	$3'' \times 3'' \times \frac{3}{8}''$	7.2		552
"	2	2.7	"	"		38.8
R. Heads	1200	—	—	24.3 per C.		290
					968	968

Total weight = 6575#

Total weight of 50 floor-beams = 328,750#

DESIGN OF THE LATERAL SYSTEMS

Top Laterals of the River Arm and Suspended Span

Of the load of 5000 lbs. at the upper chord point, one half will be assumed to be taken by the upper laterals and the other half to be transmitted by the intermediate posts to the lower lateral system.

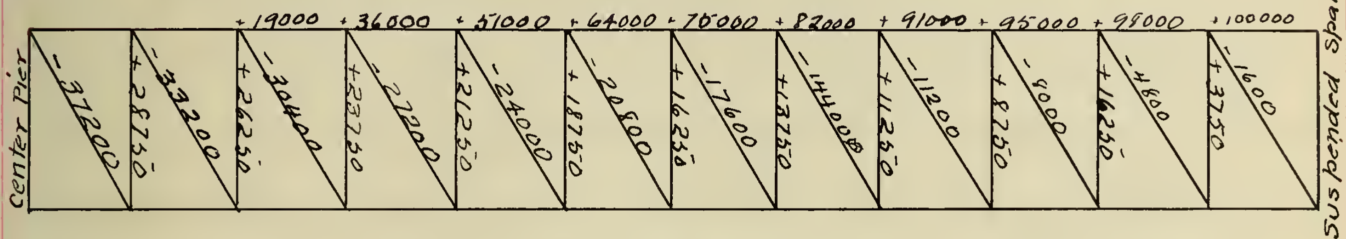
The question arises, where will the wind stresses be transferred from the upper lateral system to the lower system? An assumption must be made as to the four possible routes. The four routes are as follows;

1. Does the portal in the suspended span act as an end post in carrying the wind stresses from the upper to the lower laterals at that point? 2. Does the sway bracing in the compression member that meets the lower chord on the center pier, act as a portal by transmitting the wind stresses from the upper system to the pier, or does it only serve as sway bracing? 3. If the compression member in 2 does not take the stress, then does the vertical post over the center pier take the stresses to the pier by flexure? 4. If the stresses does not travel any of the three routes just referred to, then does the portal and end post at the shore arm pier take all the stress. Of these four routes, a combination of the 2d, 3d and 4th will be used. The sway bracing in the post over the center pier will be designed so as to act as a portal for half the bridge, in case the compression member that meets the lower chord on the center pier does not take the wind stresses from the upper laterals to the pier at that point. One half the wind on the shore arm will be taken to the shore-arm pier by the end post, and the other half will be transmitted to the center pier. The portal in the suspended span will not be designed to take the wind stresses from the upper to the lower at that point, but will serve as sway bracing only, and as a means of stiffening the post.

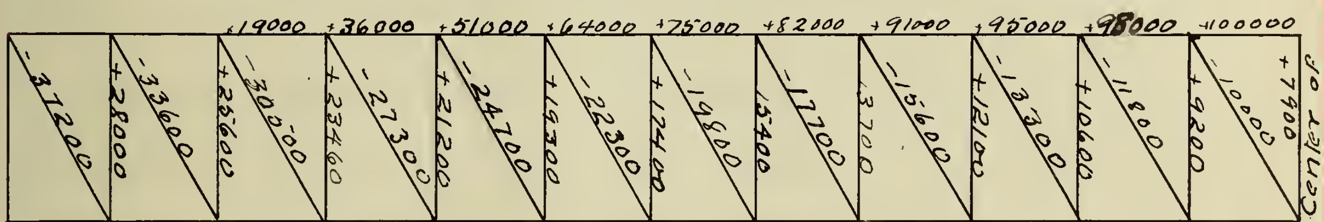
STRESSES IN THE UPPER LATERALS OF THE RIVER ARM AND ONE HALF THE SUSPENDED SPAN.

WIND CONSIDERED AS A DEAD LOAD.

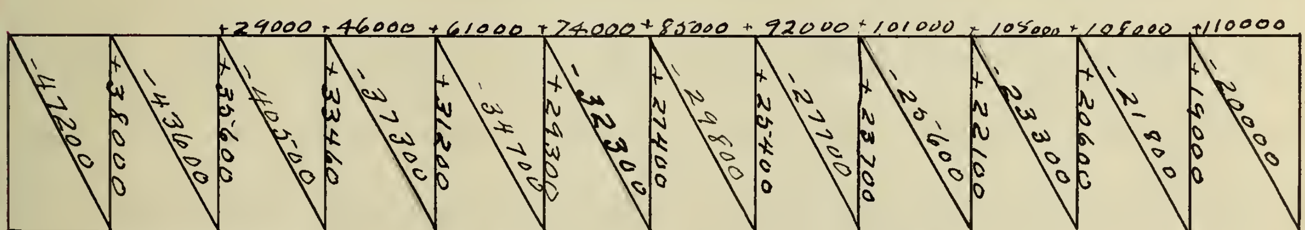
Joint Load 2500#, sec α = 1.3, tan α = 0.8.



WIND CONSIDERED AS A LIVE LOAD

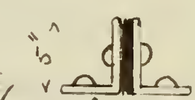


MAXIMUM STRESSES INCLUDING 10000# INITIAL TENSION.



< Laterals = 2 L's 4" x 5" x 3/8" × Laterals = 2 L's 3" x 4" x 3/8" >

The tension members will be of angles, so constructed as to take compression as well as tension, riveted together at all intersections, and will be riveted to the joint plate of the upper chord directly over the intermediate posts. Round tension members could have been more economically designed and more advantageously fastened so as to prevent eccentricity, but could not be fastened together easily at their intersection, and hence would have been less rigid. The tension allowable will be 20,000 lbs. per square inch, which conforms with Lewis's Specifications, after adding 25% on account of the wind.

Design of Tension Laterals. The tension laterals being designed to take compression in somewhat in excess, but theoretically they will take but little compression and serve only to stiffen the structure. The angles will be placed back to back (see sketch in margin) and stiffened with fillers to increase its servicability as a strut. 

The two similar members in the same bent will be placed with their compression legs in opposite directions, so as to join at their intersection. The tension laterals shall be punched $\frac{1}{8}$ inch short so that tension may be placed in them in erection, by pulling the rivet holes into line with a drift pin.

Lateral Struts. All compression members will be of the same size, for two reasons; 1, extra cost and

complication in ordering; and 2, less time will be consumed in assembling. These struts also serve as struts for the sway bracing, and will be designed heavier than is needed as a lateral strut. These members will be designed for the maximum stress, which occurs over the center pier.

Maximum Stress = 38000 lbs.

$$\text{allowable Unit Stress} = \frac{c}{1+\frac{(336)^2}{(36000)(1.9)^2}} = 5340 \text{ lbs. per sq. inch.}$$

$$\text{area required} = \frac{38000}{5340} = 7.1 \text{ square inches.}$$

Try two $3'' \times 5'' \times \frac{3}{8}''$ angles having an area of

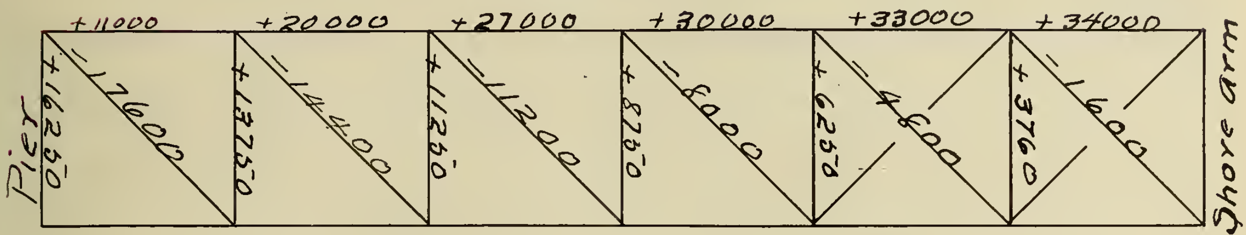
area	=	11.44	square	inches.
Subtract for rivets	=	4.00	"	" .
Net area	=	7.44	"	" .

$$\text{Efficiency} = \frac{7.44}{7.1} = 105\%.$$

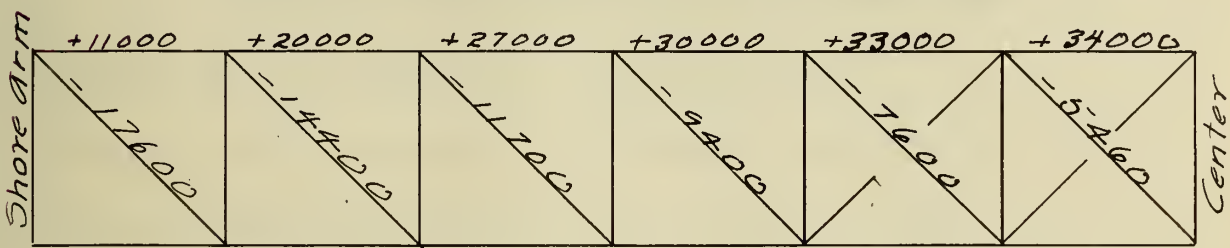
Top Laterals of Shore Arm. The wind in the upper chord of the shore arm was considered as going both to the shore arm and to the center pier equally. The wind was considered both as a live and dead load.

STRESSES IN UPPER LATERALS OF SHORE ARM

WIND CONSIDERED AS A DEAD LOAD

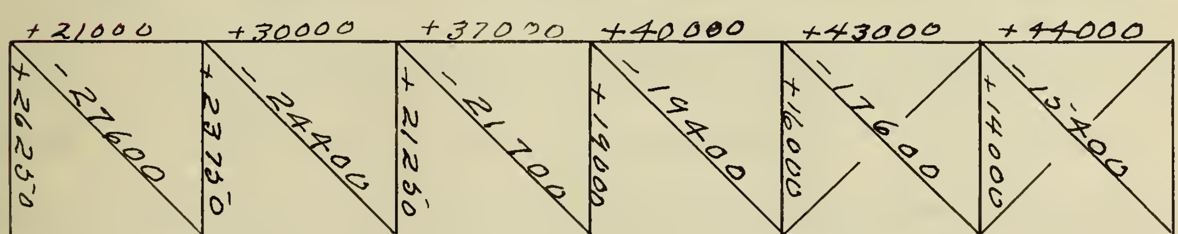


WIND CONSIDERED AS A LIVE LOAD



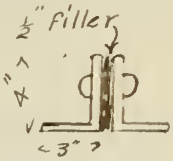
MAXIMUM STRESSES

INCLUDING 10000# INITIAL TENSION



Design of tension members. All tension members will be designed stiff, so as to take compression as well as tension.

Assume two $3'' \times 4'' \times \frac{1}{2}''$ angles.



Gross area	= 6.5	square inches.	
Subtract for rivets	2.0	"	"
Net area	= 4.5	"	"

Maximum Stress = 27,600 lbs.

Area required = $\frac{27600}{20,000}$ = 1.38 square inches.

This makes a high efficiency, but no smaller angle can be used economically, since a smaller angle would be useless in compression

Compression Struts. All struts will be the same as in the river arm, for reasons as given on page 29.

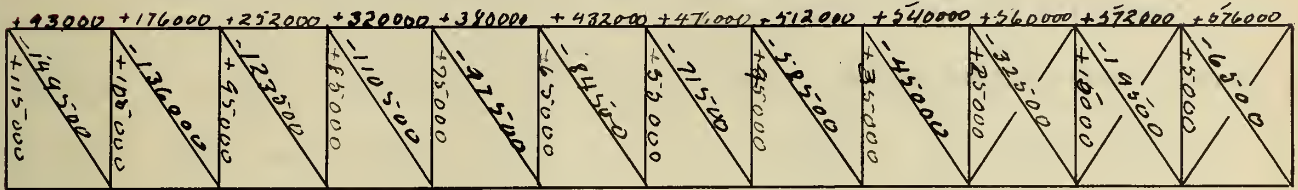
The compression in the upper chord of the vertical truss due to the wind on the lateral truss will not affect the design of the upper chord, since the wind stresses do not approach 25% of the dead load stress.

Lower Laterals of River Arm and Suspended Span.

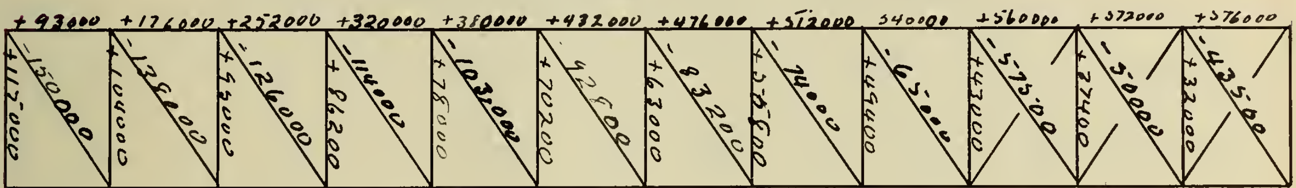
The wind stress carried by the lower laterals is equal to 200 pounds per linear foot of bridge plus 100 pounds carried down the intermediate posts and 100 pounds per linear foot of bridge for the wind pressure on the live load. This wind pressure is excessive (see page 10-11), but the provision for this wind makes a structure stable against oscillations due to the live load.

STRESSES IN THE LOWER LATERALS OF THE RIVER ARM AND ONE HALF THE SUSPENDED SPAN

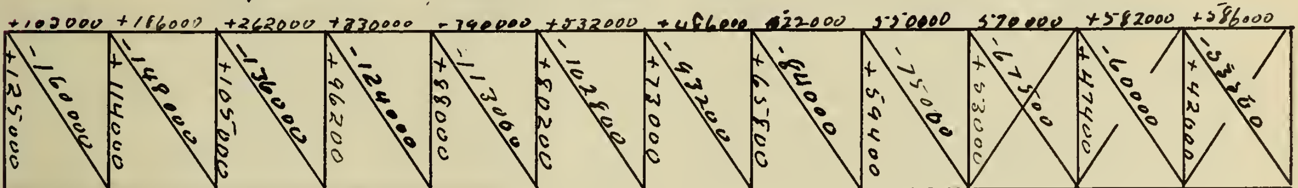
WIND CONSIDERED AS A DEAD LOAD



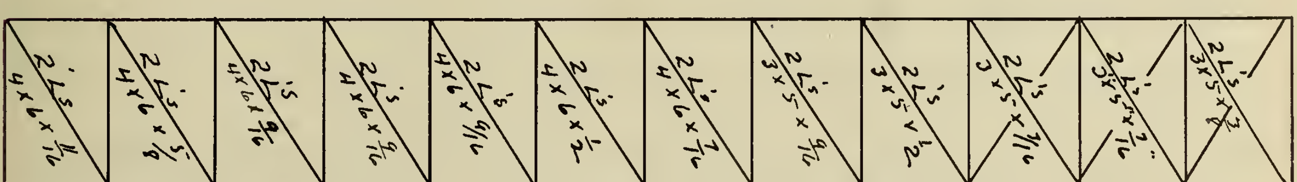
WIND CONSIDERED AS A LIVE LOAD



MAXIMUM INCLUDING 10000# INITIAL TENSION



SIZES OF TENSION DIAGONALS

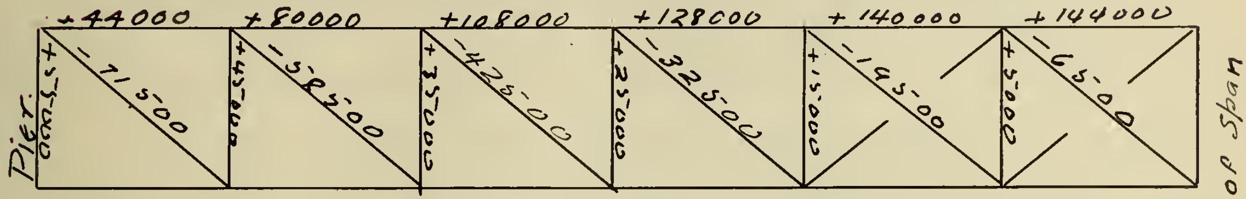


Design of tension members. The allowable stress = 20000# per square inch for tension, and 16000# per square inch for compression, and as the angles will be designed for compression as well as tension, 16000# will there-fore be used. The tension diagonals are fastened to plates on the bottom of the floor beams in the plane of the lower chord pin. The bottom of the stringers, excepting those of the side-walks, will be in line, so that the angles can be riveted at their inter-sections. Since the bottoms of the stringers are in line, and since they are not of the same depth, a timber piece will be placed upon the top of the smaller, upon which rests the floor.

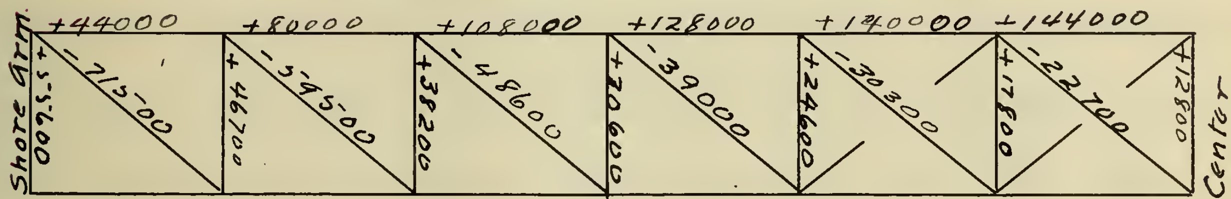
Compression Members. The bracing of the floor-beams by the stringers will make the floor-beam rigid enough to carry the compression of the lower laterals as a strut, and hence no special floorbeam or strut will be needed to take this compression. Extra struts will be required in the cantilever arms, where the line of the pins is located beneath the floor-beam level. These struts will have to be exceedingly heavy, and will require a special design. In ad-dition to their being struts for the lower lateral system, they will serve the same purpose for the lower sway bracing, and will have to be designed with a high efficiency to take these stresses.

STRESSES IN THE LOWER LATERALS OF THE SHORE ARM

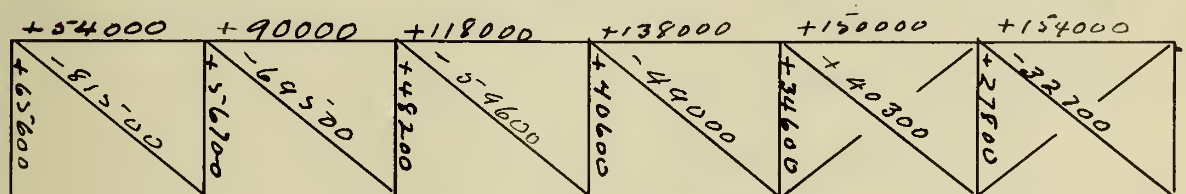
WIND CONSIDERED AS A DEAD LOAD



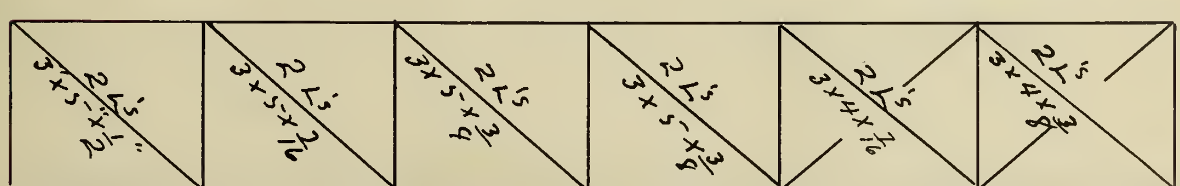
WIND CONSIDERED AS A LIVE LOAD



MAXIMUM INCLUDING 10000# INITIAL TENSION



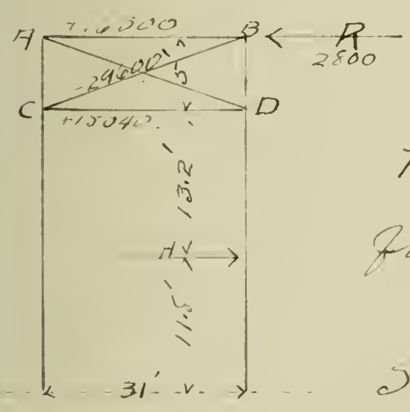
SIZES OF TENSION DIAGONALS



SWAY BRACING.

The sway-bracing can not be designed very accurately, but is generally made very strong for rigidity. The stresses in the sway-bracing are due to the wind pressure on the post, while in the upper compression strut are these plus the stresses from the upper laterals, for which it also serves as a strut, and for which it will also be designed. Sway bracing will also be placed below the floor beams in the panels near the center pier, where the distance between the pin line and the floor beam line differ by more than 5 feet. The sway bracing in the long intermediate posts will not have their struts more than 25 ft. apart.

SWAY BRACING IN THE SUSPENDED SPAN.



$R = 2500 + 300 = 2800^{\#}.$

$H = \frac{1}{2} R = 1400^{\#}.$

Plane of Contraflexure $= \frac{25}{2}\left(\frac{25+60}{50+30}\right) = 11.8$ ft. from the lower pin line.

$V = \frac{(2800)\,30}{31} = 2750^{\#}.$

Stress in $AB = 2800 + \frac{(1400)(13.2)}{5} = +6500^{\#}.$

Stress in $CD = \frac{(1400)(18.2)}{5} = +5040^{\#}.$

Stress in AB and $BC = V \sec \alpha = 14600^{\#}.$

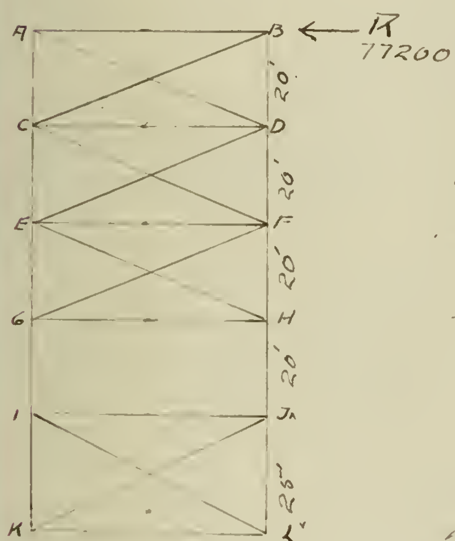
The sway bracing will be designed to take $10000^{\#}$ initial tension, for which see cut for the final stresses. The tension members will be cylindrical rods, the initial stress being put in them with turn-buckles.

Sections of AD & CB $= \frac{21600}{20000} = 1.5$ square inches.
which will require a $1\frac{3}{8}''$ cylindrical bar.

Section of AB;- as a strut stiffening laterals, and also located in the center of the bridge - has an efficiency of several hundred per cent, and hence will not be investigated further.

Section of CD: Allowable Stress $= \dfrac{c}{1 + \frac{(336)^2}{(36000)(1.9)^2}} = 5340$ # per square inch for which assume two $4'' \times 6'' \times \frac{1}{2}''$ angles having 9.5 square inches area. Area required is $\frac{15040}{5340} = 2.8$ square inches, hence the efficiency is 330%.

SWAY BRACING FOR POST OVER CENTER PIER.



The posts were assumed as two ft. wide, which is the width in the St. Lawrence River bridge. Consider the maximum condition; viz.; that the wind stresses in the river arm and one half the suspended span, and one half the shore arm are carried down the post over the center pier by flexure.

$R = (300 + \frac{325}{2})100 + (40)30 = 77200$ lbs.

The plane of contraflexure was taken as one half the height CK, by considering the portion of the sway-bracing above C to act as a portal.

$H = \frac{R}{2} = 38600$ #.

Stress in AB $= \dfrac{(77200)20 + (38600)20}{(2700)60} = 139100$ #.

Stress in CD = $\frac{(38600)60 - (2400)40 - (2700)80}{20}$ = 100,200#.

" " AD = $\frac{(77200)20}{31}$ sec i = 89,560#.

" " CF = $\frac{(77200)40 + (2400)20}{31}$ 1.8 = 101,000#.

" " EF = $\frac{(77200)60 + (2400)60}{20}$ = 239,700#.

" " GH = $\frac{(77200)80 + (2400)120 - (38600)20}{20}$ = 284,6[illegible]#.

" " EH = $\frac{(77200)60 + (2400)60}{31}$ = 159 200#.

" " IJ. = (77200)105 + (2400)(195) + 2700 − (38600)(45) = 344,550#.

" " IL = $\frac{344550}{31}$ sec i = 19,900#.

Design of Sections.

GH. Allowable = $\frac{16000}{1+\frac{137000}{(78)23}}$ = 12500# per square inch.

area required = $\frac{284600}{12500}$ = 23 square inches.

Use two channels 15" 40# with area of 23.52 sq. in.

EF. allowable unit stress is same as before.

area required = $\frac{239700}{12500}$ = 19.2 square inches.

Use two [s 15" 35# with area of 20.58 sq. in.

AB. unit stress same as GH.

area required = $\frac{134100}{12500}$ = 12 square inches.

Use two [s 12" 25# having area of 14 sq. in.

AD. unit stress 20000.#

area required = $\frac{89560}{20000}$ = 5.0 sq. inches.

Use two 4"×6"×$\frac{9}{16}$" Ls having gross area of 10.62 sq. inches, from which subtract for rivets 2.00 sq. in.

Net area of section = 8.62 sq. in.

CF. Area required = $\frac{101000}{20000}$ = 5.25 sq. in.

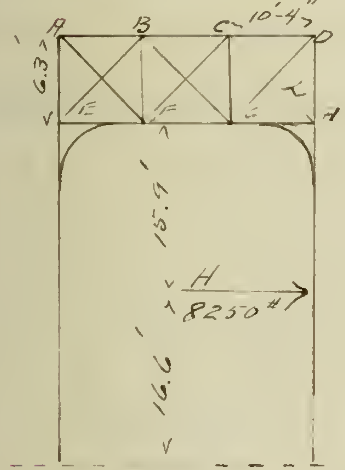
Use two 5"×3"×$\frac{1}{2}$" Ls having gross area of 7.5 sq. inches, subtracting 2.00 for rivets leaves 5.5 sq. in.

EH. Unit Stress = 20000#. Area required = $\frac{159200}{20000}$ = 8 square inches. Use two $4'' \times 6'' \times \frac{9}{16}''$ L^s having a gross area of 10.62, subtracting from this 2.0 for rivets leaves 8.62 square inches.

These tension angles will be connected by a plate at their intersection, and be riveted together at intervals, by using in addition fillers, so as to take compression as well as tension.

DESIGN OF PORTALS.

In designing the portal attention will be paid to the appearance, as well as to the strength.



$R = 15000^{\#}$

$\frac{P}{2} = 1500^{\#}$

$R + \frac{P}{2} = 16500^{\#}$

$H = \frac{1}{2}(16500) = 8250^{\#}$

$V = \frac{(16500)(39)}{31} = 20800^{\#}$

Plane of contraflexure is =

$\frac{32.5}{2}\left(\frac{32.5+78}{65+39}\right) = 16.6$ ft. from bottom.

The end post will be considered fixed, as will be proven later.

Stress in $CD = 16500 + \frac{(16500)15.9}{6.3} = 58,000^{\#}$

" " $GH = \frac{(16500)(22.2)}{6.3} = 58000^{\#}$

" " $CH = V \sec.\alpha = 20800 \sec. 58° - 40' =$

$(20800)\,1.92 = 40,000^{\#}$

Sections.

AB: stress in the member = $58000^{\#}$

Allowable stress = $\frac{16000}{1+\frac{(370)}{(18000)37}} = 12300^{\#}$ per square inch.

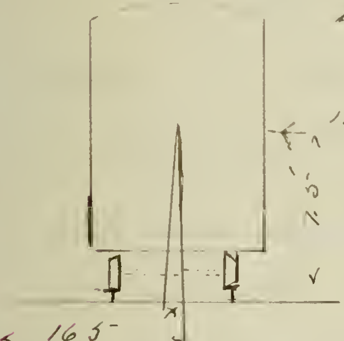
area required = $\frac{38000}{12300}$ = 5 square inches.
assume two 10" [^s @ 15# with area of 8.42 square inches. Efficiency 178%.

CH: Stress 40,000 lbs. - [illegible] + initial tension. allowable unit stress 20,000#. area required = 2. square inches, requiring $1\frac{1}{2}$" cylindrical bar. This bar will be supplied with turnbuckles to place them in initial tension.

EH. This member will be the same crossection as AD as they have the same stress.

STRESSES IN TRUSS MEMBERS DUE TO WIND.

The estimated exposed area of a 25 ft. car is 7 x 17 = 119 square feet. This estimate leaves 4 ft. at each end of the car for the platform, and coupling room.



Wind pressure per foot of bridge equals

$$\frac{120 \times 30}{25} = 150\text{\# approximately.}$$

The weight of the car is nearly 1000# per foot.

The per cent more carried by the truss on the lee ward side over the one on the wind ward side = x : 31 :: 150 : 1000, which makes x 4.5 feet or a percentage of 15. This 15% is of the live load only, and as the dead load on the truss exceeds the live load, the actual per cent of both is only 6.6% which can be neglected, since an excess of 25% is allowable.

TRUSS MEMBERS

The trusses will be designed for the following unit stresses.

Tension: Rolled eye-bars (truss members) $= 12000(1 + \frac{min}{max})$.
Plates and shapes $= 11000(1 + \frac{min}{max})$.

Compression: Two square bearings $= R = \frac{c}{1+\frac{l^2}{36000 r^2}}$.
One square and one pin $= R = \frac{c}{1+\frac{l^2}{24000 r^2}}$.
Two pin bearings $= R = \frac{c}{1+\frac{l^2}{18000 r^2}}$.

in which R = allowable stress in pounds per square in.

l = length in inches.

$C = 10000(1 + \frac{min}{max})$.

r = least radius of gyration.

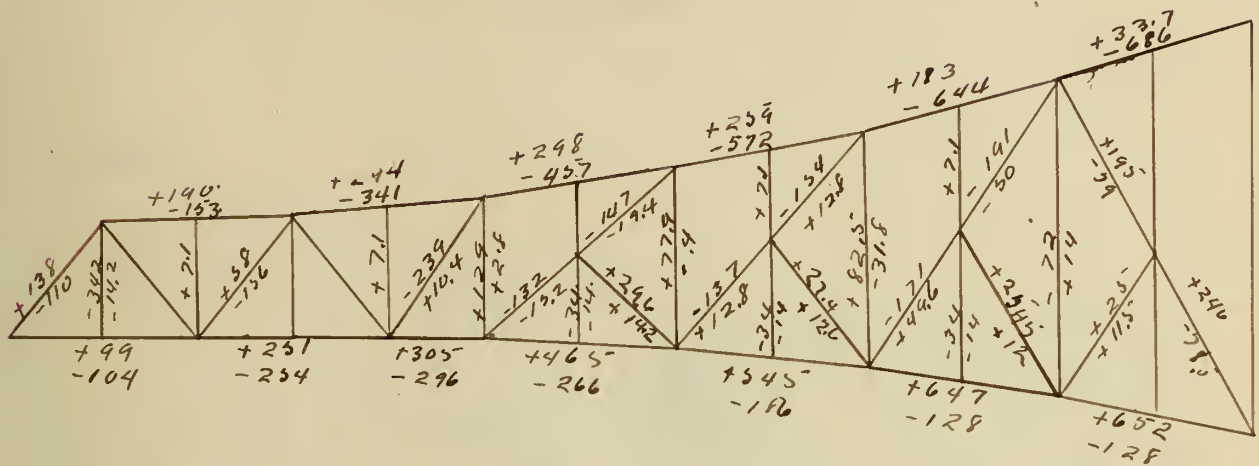
Alternate Stresses use the following;

Tension $= 11,000(1 - \frac{1}{2} \cdot \frac{s}{S})$.

Compression $= 10,000(1 - \frac{1}{2} \cdot \frac{s}{S})$.

in which S and s are the larger and smaller stresses respectively. In designing the members having alternate stresses, in which the compression is the greater, the value of $10000(1 - \frac{1}{2}\frac{s}{S})$ will be used in the formulas for R in place of C. For the members in which the tension stress exceeds the compression, the allowable unit stress found is used in place of R.

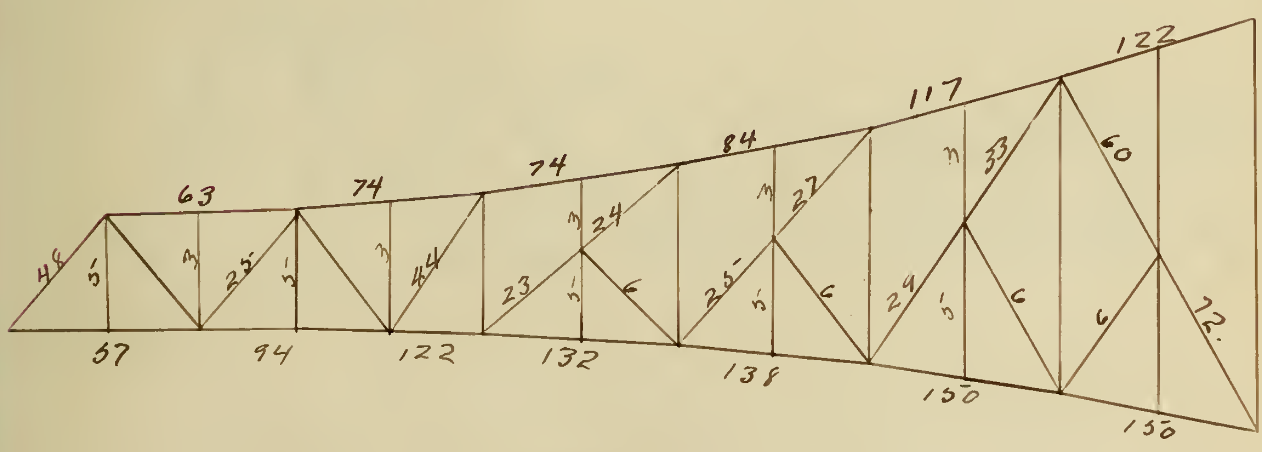
MAXIMUM AND MINIMUM STRESSES IN SHORE ARM IN TONS



AREAS OF CROSS SECTION OF THE SHORE-ARM

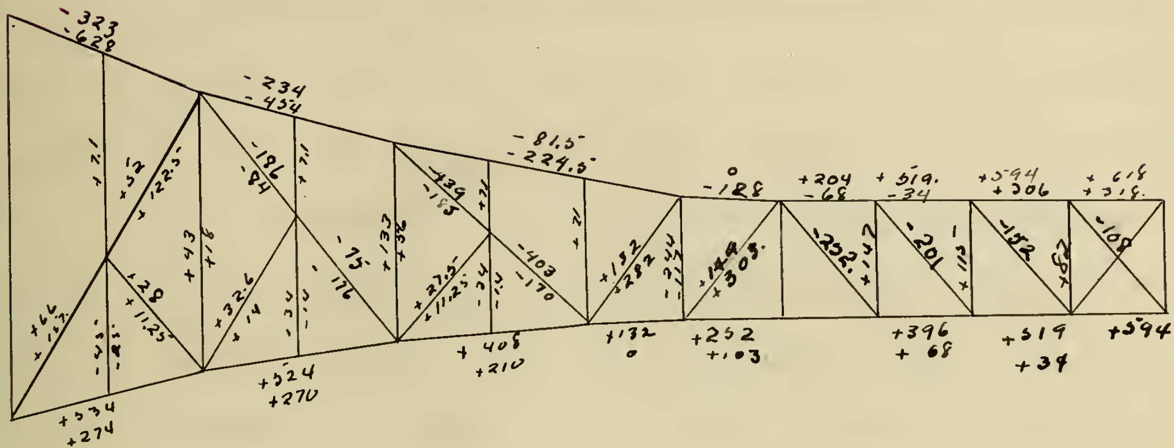
In determining the first approximation of the area of the upperchord the depth assumed was 25"

Areas in square inches



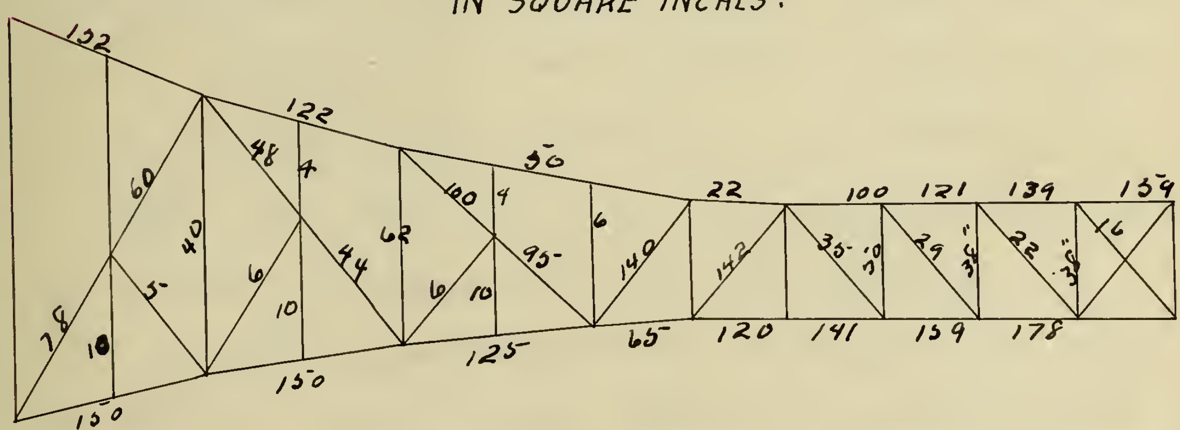
MAXIMUM AND MINIMUM STRESSES IN THE RIVER ARM AND SUSPENDED SPAN

IN TONS



AREA OF CROSS SECTIONS

IN SQUARE INCHES.

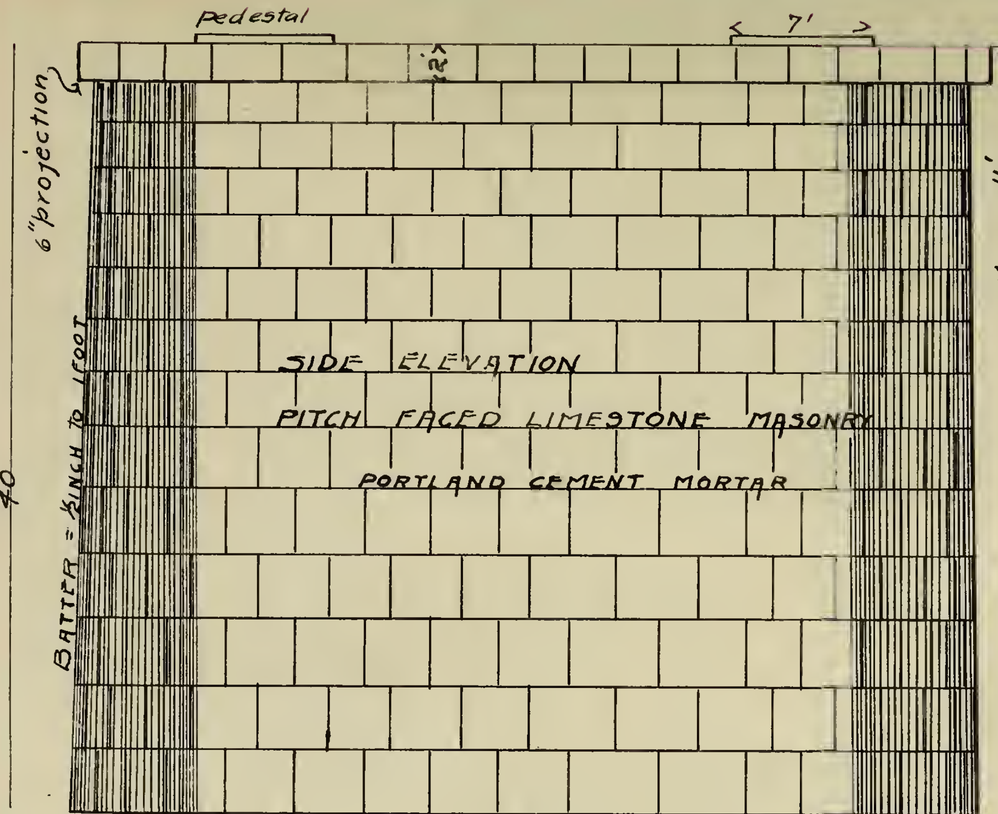


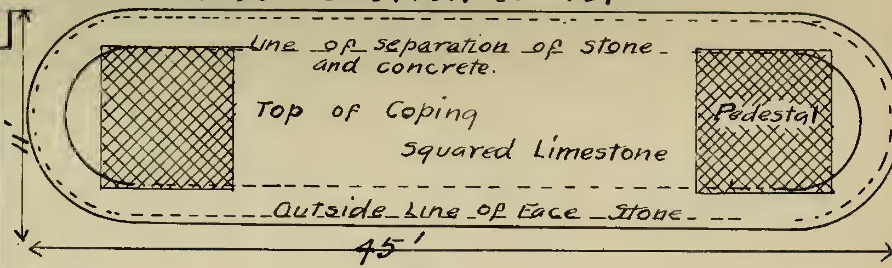
DESIGN OF THE CENTER PIERS

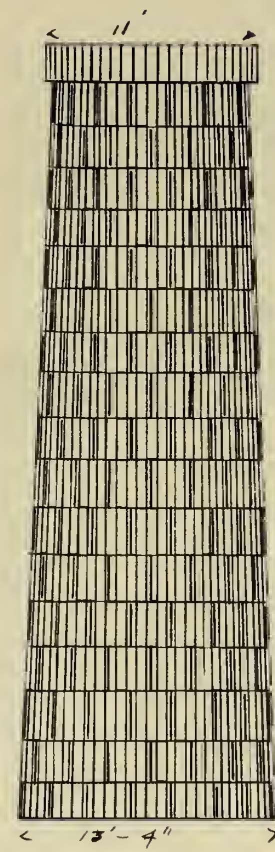
The center pier is shown on page 44. The attempt was to make an economical as well as an artistic design. The pier proper will be constructed of concrete, since concrete has in late years been an important factor in all masonry work. The concrete pier faced as smoothly as possible, would be sufficiently artistic for an unexposed site, but for a city it was considered too crude and unsightly. The concrete core will be protected from the ravages of the current, and from public inspection by a 2-foot face of pitch faced limestone. The ashlar facing will be fastened to the concrete core by steel hooks. The concrete core was designed to take all the weight of the bridge and its load, the facing being put on for effect. The concrete core was adapted on account of the excessive weight of the bridge, its mobility, or the ease of building an artistic shape, the use of inferior sizes of stone, and the opportunity of using non-skilled labor.

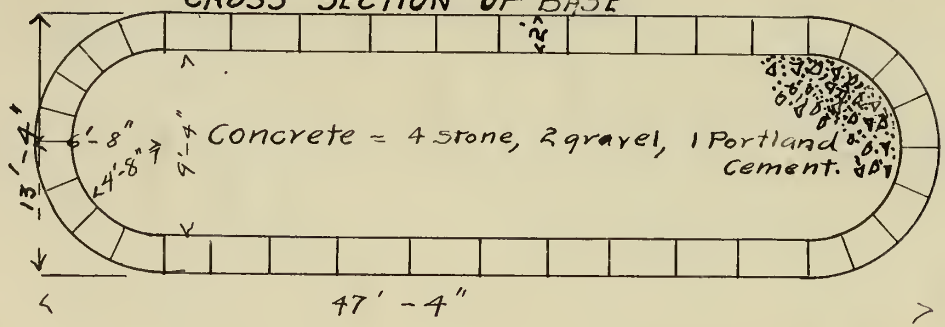
The height was estimated at 40 ft. The length at the top is 45 ft, which allows 3.5 ft. outside of the outer edge of the pedestal. The batter is $\frac{1}{2}$ inch to the foot.

A pressure of 300 lbs. per square inch was allowed for the bearing on the masonry, thus making the pedestals 7 ft. square. Anchor bolts will be placed to at least a depth of 6 ft. The concrete core and facing will be covered by coping 2 ft. thick and will project 6 inches.









DESIGN OF CENTER PIERS

SCALE 1 IN. = 10 FT.

CONTENTS

IN TWO PIERS

LIMESTONE MASONRY

628.13 CU. yds.

CONCRETE = 897.4 CU. yds.

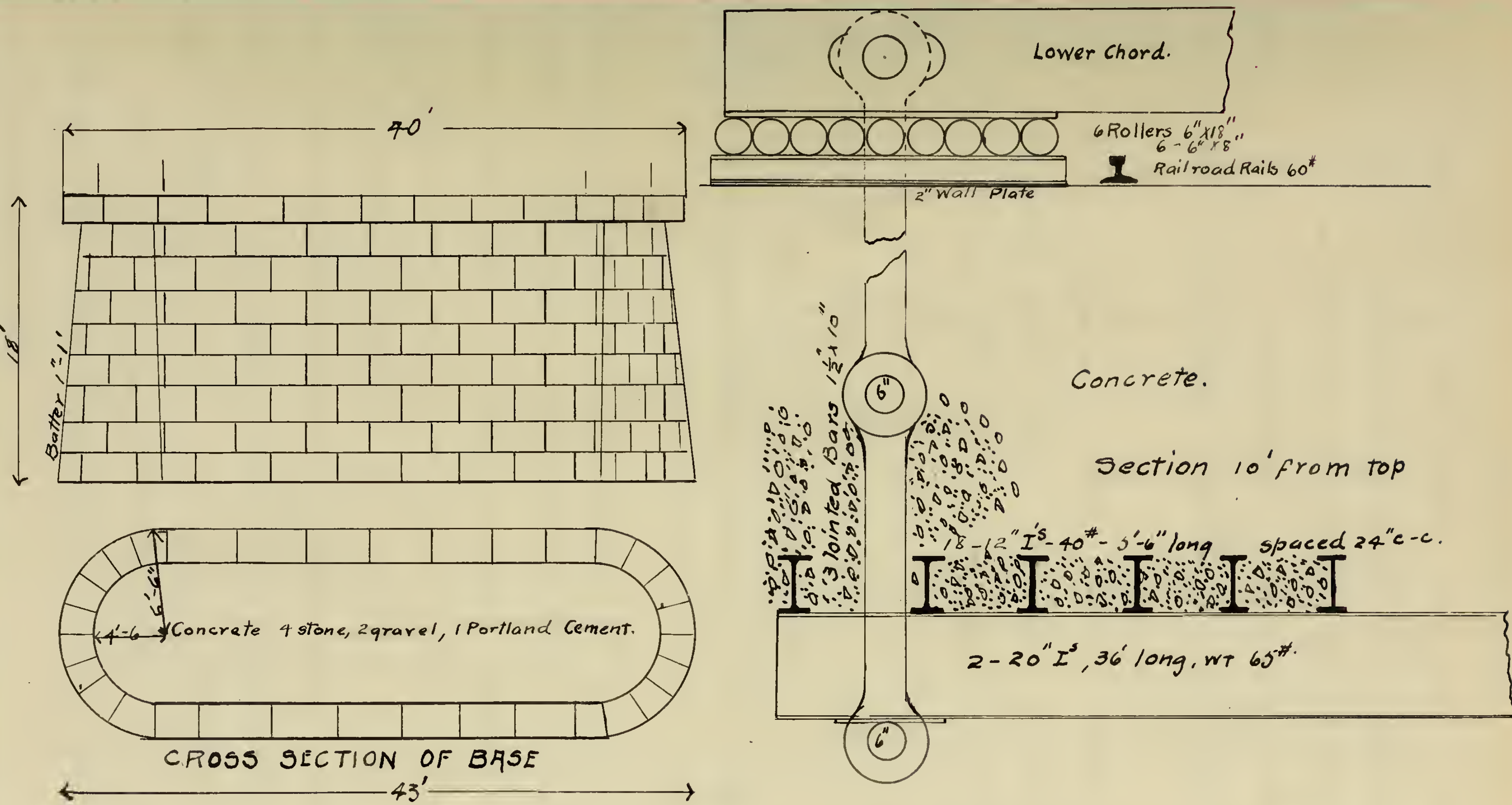
DESIGN OF THE SHORE ARM PIERS

The shorearm pier is shown on page 46. It will be constructed with a concrete core and a limestone face, similiar to the center piers.

Waddell gives a general rule, that the anchor piers in a cantilever bridge shall have a weight after considering its bouyant effect, equal to at least twice the maximum tension in the anchorage. The maximum tension on the anchor bar occurs when the river arm and one half the suspended span is loaded with the live load. This maximum tension amounts to 180 tons, which limits the weight of the pier to 360 tons.

The length of the pier will be 39 ft., thus allowing 4 feet outside the center line of the truss for the pedestal and the projection; and the width will be 6 feet. Hence a pier of these dimensions to have a weight of 360 tons must have a depth of at least 8 ft. on top of the steel I's to which the tension bar is attached. The life of the steel I's depends on the absence of moisture and with this fact in mind the depth under the steel was made 8 feet, which makes a pier 18 ft. high to the top of the coping. The batter allowed will be 1 inch to the foot.

Special care should be taken with the connection to the bridge by the anchorage bar, so as to have the bar-shaft absolutely water tight, since the life of the anchorage depends on this one point.



DESIGN OF SHOREARM PIER
AND CONNECTIONS

EXPANSION.

The expansion of the bridge was figured for a temperature change of 150° Fahrenheit, using the co-efficient of linear expansion as .0000065. The expansion of the two shore cantilever arms is 0.31687 ft. each, and of the center span 0.555 ft. — making a total linear expansion of 1.18274 ft.

The expansion will be taken at two places, viz; the two shore arm piers, and to aid this arrangement rollers will be placed on one of the center piers. The expansion in the center span could have been taken by hanging the suspended span to the river cantilever arms, and providing slotted holes. This is a simple and common method, but was not adapted for four reasons, viz; 1 at the best it makes a very cumbersome, unsightly joint with questionable efficiency; 2. with a joint of this kind both the upper and lower chord members of the river cantilever arm require a special design, and pieces must be inserted temporarily for the purpose of taking the erection stress only; 3. it is an exposed joint and its parts rust badly; 4. the last and most important is that it would make the center span less rigid. The expansion at the shore arm pier remote from the fixed center pier is 0.87187 ft. and that at the shore arm pier nearer the fixed center pier is 0.31687 ft.

Rollers on the Center Pier. The maximum reaction at the center pier due to the dead weight on the truss = $21.25\left(\frac{625}{2}\right)$ = 9 times the reaction at the shore-arm pier due to the dead load = 530.25 tons. The maximum reaction at the center pier due to the live load on the truss = 500 tons.

Total Reaction = 1031.25 tons = 2,062,500 #

Assume rollers 10 inches in diameter. The allowable load per linear inch of roller = $1000\sqrt{d}$, where d is the diameter in inches = 3160 lbs. This gives a length of roller of $\frac{2062500}{3160}$ = 652 inches. Assuming rollers 6 ft. long, the bridge would require $\frac{652}{72}$ = 9+ or say 10. Hence 10 rollers 6 ft. long and 10 in. in diameter will be used. This requires a bed plate 7 x 8 ft.

Rollers at the end of the shore arm. The maximum pressure on the shore-arm pier occurs when the shore-arm is covered with the live load, and is equal to $6 \times 20 + 9 = 129$ tons, or 258,000 #. Assuming 6 in. rollers, for which by the formula the allowable stress is 2450 lbs. per linear inch, gives a length of roller $\frac{258000}{2450}$ = 101 inches. Assuming 18" rollers, the number required = $\frac{101}{18}$ = 6. It is probable that some of these will have to be cut to allow room for the connection of the expansion device to the anchorage, and therefore 8 will be used.

ADJUSTING DEVICES.

It is necessary to provide some means of varying the length of the channel-span to correct for any errors in the location of the end pedestals of the shore arms. This is accomplished by means of a temporary device, by means of which the bridge is connected and adjusted, and after such adjustments are removed. No description will be given of these devices, because they are so complicated that only a drawing will suffice, but on account of the time at hand no drawing will be made.

CAMBER.

Camber will be provided by lengthening or shortening the several members equal to their distortion under the maximum live and dead loads. Half the camber will be taken out in the connection of the floor-beam to the post.

ERECTION

The erection of the shore cantilever arms will be similar to that of common bridge erection. The shore-arm piers must be very carefully located, because upon these depends the location of the center piers, and also the accuracy of the connection in the center of the suspended span. The piers will be located by a system of triangulation

The center span will be erected by two traveling

derricks, each weighing 40 tons, including ballast. The derricks will erect ahead of themselves from the center piers outward to the center of the suspended span, the steel delivered to them from scows in the river.

COST.

It is not possible to make any exact estimate of the proposed structure, without having completed the details of the design. The time at command does not justify a further elaboration of the design.

Without the detailed estimate of the cost of the proposed design, it is not possible to make any definite comparison between the cost of the proposed cantilever bridge and the present structure. Farther, the two bridges differ materially in their essential features. The cantilever bridge has a 28 ft. roadway, while that of the present structure is 20 ft. The cantilever bridge was designed as a class A bridge, while the present structure is Class B. The proposed bridge was designed to carry a heavy electric car traffic, while the existing bridge carries none. The cantilever bridge has steel joists under both the roadway and sidewalk, while the present bridge has wood joists. Farther, shortly before, if not at the time the present structure was built, steel manufacturers were in a pool, and the price of bridge material

was about twice that current at present.

The contract price for the present structure, including masonry and foundations is reported to have been $42,500. It is probable that a cantilever bridge to fulfill the same conditions would cost considerably more than this — at least 25%, probably 40%, and possibly 50%.

CONCLUSION.

The site is not one favoring the cantilever type. 1. Bed rock is near the surface and therefore the foundations are comparatively inexpensive. 2. Stone may be obtained from a quarry near-by, and therefore masonry is cheap. 3. The stream is not navigable, and therefore a clear water-way is not important. 4. The high water flow is small in comparison with the area of the cross section between bluffs, and therefore a considerable obstruction of the high water flow is unimportant.

However, even though the proposed design is unsuitable for the site, the writer feels repaid for the time he has expended upon it.